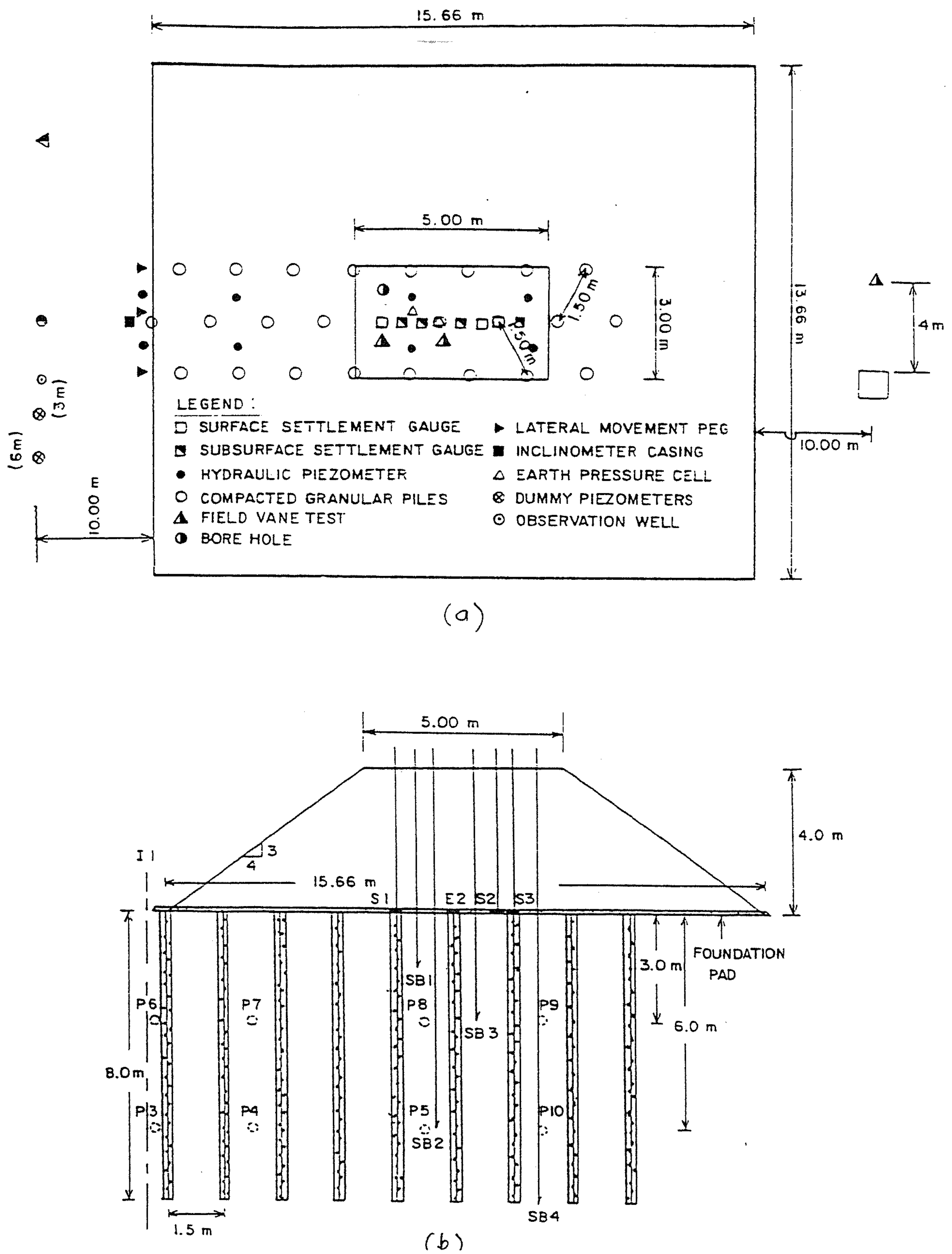


Fig. 5.17 Plan and Embankment Cross-Section Including Locations of Instrumentations, Field Tests and Field Sampling

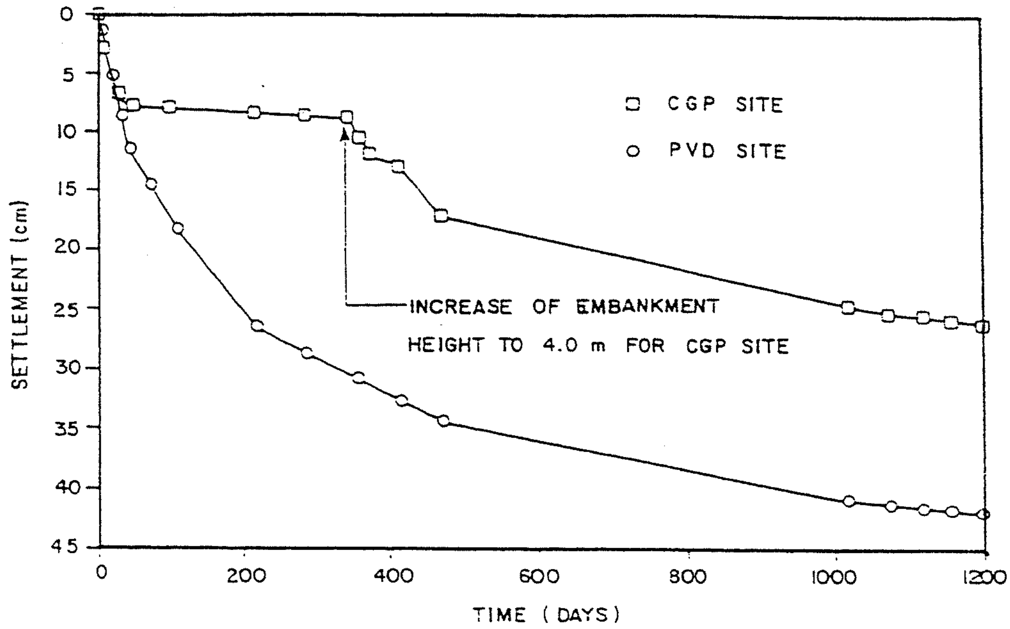


Fig. 5.18 Comparison of Maximum Surface Settlements at Granular Piles and Vertical Drains Test Embankments (Bergado et al. 1990a)

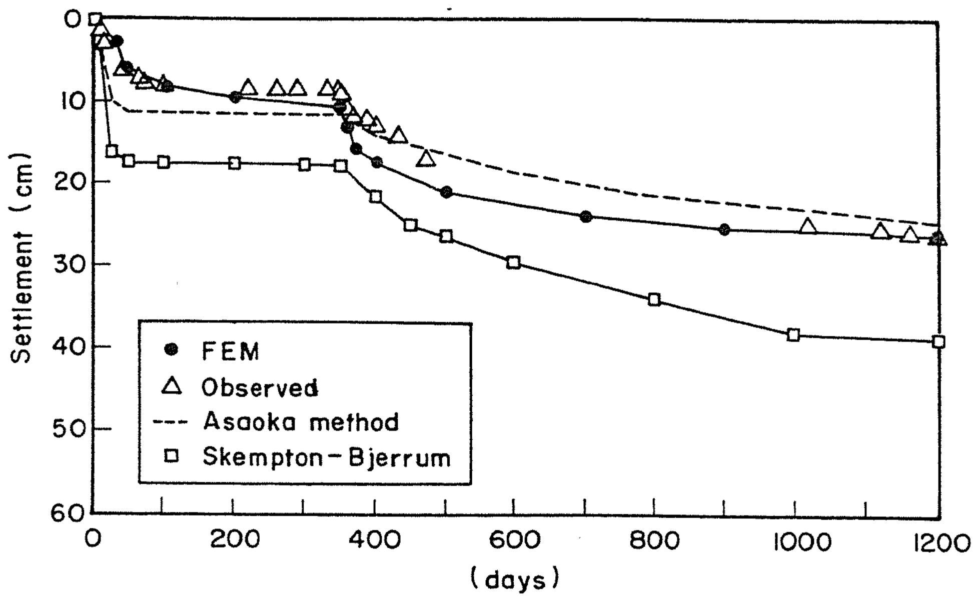


Fig. 5.19 Comparison of Primary Settlement at Embankment with Granular Piles

effects of ground subsidence caused by pore pressure drawdown (see Fig. 4.37) by axi-symmetric model. For the two-dimensional case, the FEM program CON2D was used (Duncan et al. 1981). In the case of the axi-symmetric model, the FEM program CONSAX was utilized (D'Orazio and Duncan, 1982). For 2-D consolidation analysis of the test embankment, the granular piles were converted into continuous granular walls with the same spacing and area replacement ratio as that of the actual case (see Fig. 4.31). The converted permeability including the smear zone is introduced based on the condition of equal discharge with the assumption that the coefficient of permeability is independent on the state of flow (see Fig. 4.32). The subsidence effects were evaluated assuming that the pore pressure drawdown due to deep well pumping is uniform throughout the improved ground and that the axi-symmetric model is valid.

The predicted and actual consolidation settlements are shown in Fig. 5.19. The results of FEM and Asaoka (1978) method agreed with the observed data. The results from the Skempton and Bjerrum (1957) method overestimated the settlements. Figure 5.20 shows the predicted settlement at different pile lengths. The total subsidence including ground subsidence effects were almost the same using 6 m and 8 m length of piles. Therefore, it is more economical to use 6 m length of granular piles down to the medium stiff clay layer (Bergado and Long, 1992).

5.14 MODEL INFRASTRUCTURE PROJECTS ON SOFT GROUND

A test excavation was constructed to study the mechanical behavior and applicability of different soil improvement methods to excavated slopes on soft Bangkok clay (Warinsisak, 1991). The soft clay properties at the site are given in Fig. 5.21. The test excavations consists of two unimproved and two improved slopes, as shown in Fig. 5.22. One of the improved slopes was treated with sand compaction piles. The instrumentation consisted of 6 inclinometers, 1 extensometer, 7 settlement gauges, 6 piezometers, and a water stand pipe. The excavation was carried first using the dry method using water jets.

A simplified compozer method was employed in the construction of sand compaction piles and was carried out before the test excavation. The steps in the construction are illustrated in Fig. 5.23. A 3.6 ton (35.3 kN) hammer was used to drive the 0.40 m diameter casing pipe into the soft ground to the designated bottom. The desired volume of sand was loaded into the bucket with a hopper using a backhoe and poured into the casing as the latter was being pulled up. The hammer, dropped from a height of about 10 cm, was used to compact the sand until the casing sunk 1.0 m making a sand compaction pile of 50 cm in length for every cycle of compaction. A photograph of the simplified compozer method equipment for installation of sand compaction piles is shown in Fig. 5.24.

During the excavation process, a crack was observed along the shoulder about 7.0 m from the top of the slope. The horizontal and vertical separation of the crack measured about 250 mm and 100 mm, respectively. The excessive horizontal displacement during the excavation process as shown in Fig. 5.25 could also indicate slope failure. The slope when the excavation

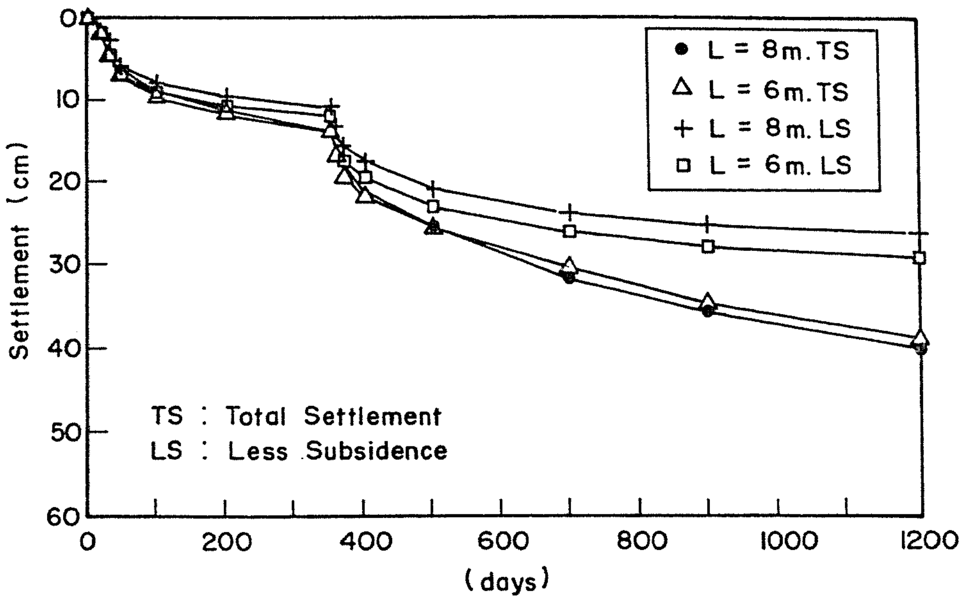


Fig. 5.20 Settlement with Differential Lengths of Granular Piles

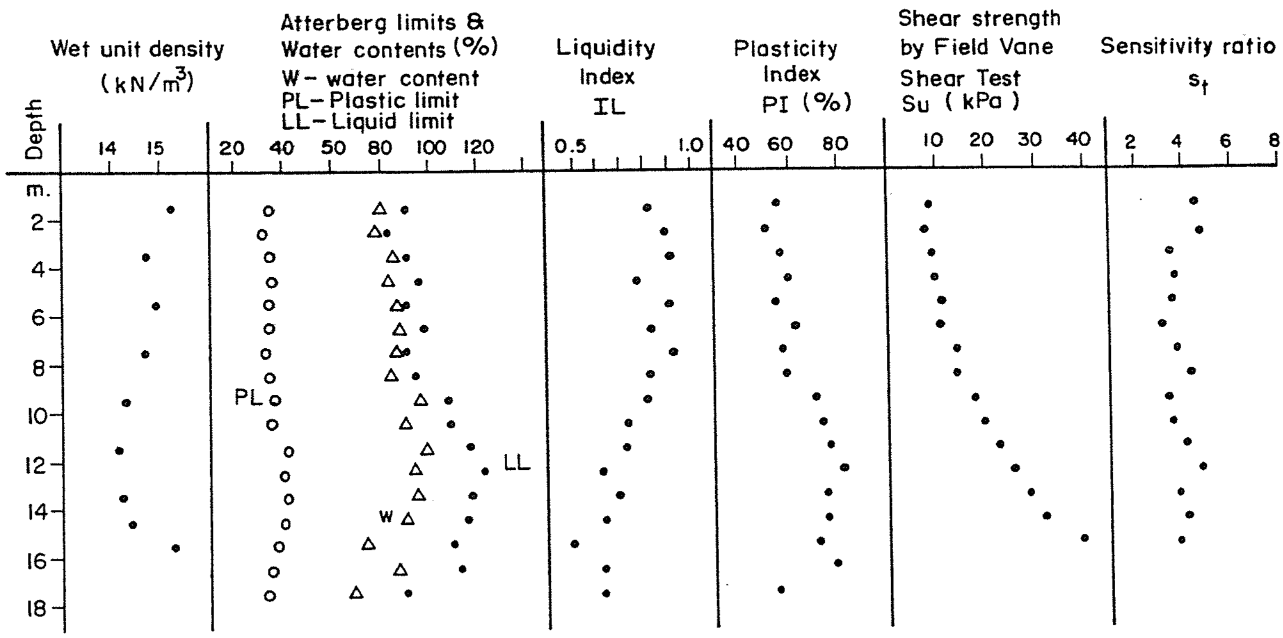


Fig. 5.21 Engineering Properties of Soft Bangkok Clay at Bang Bo

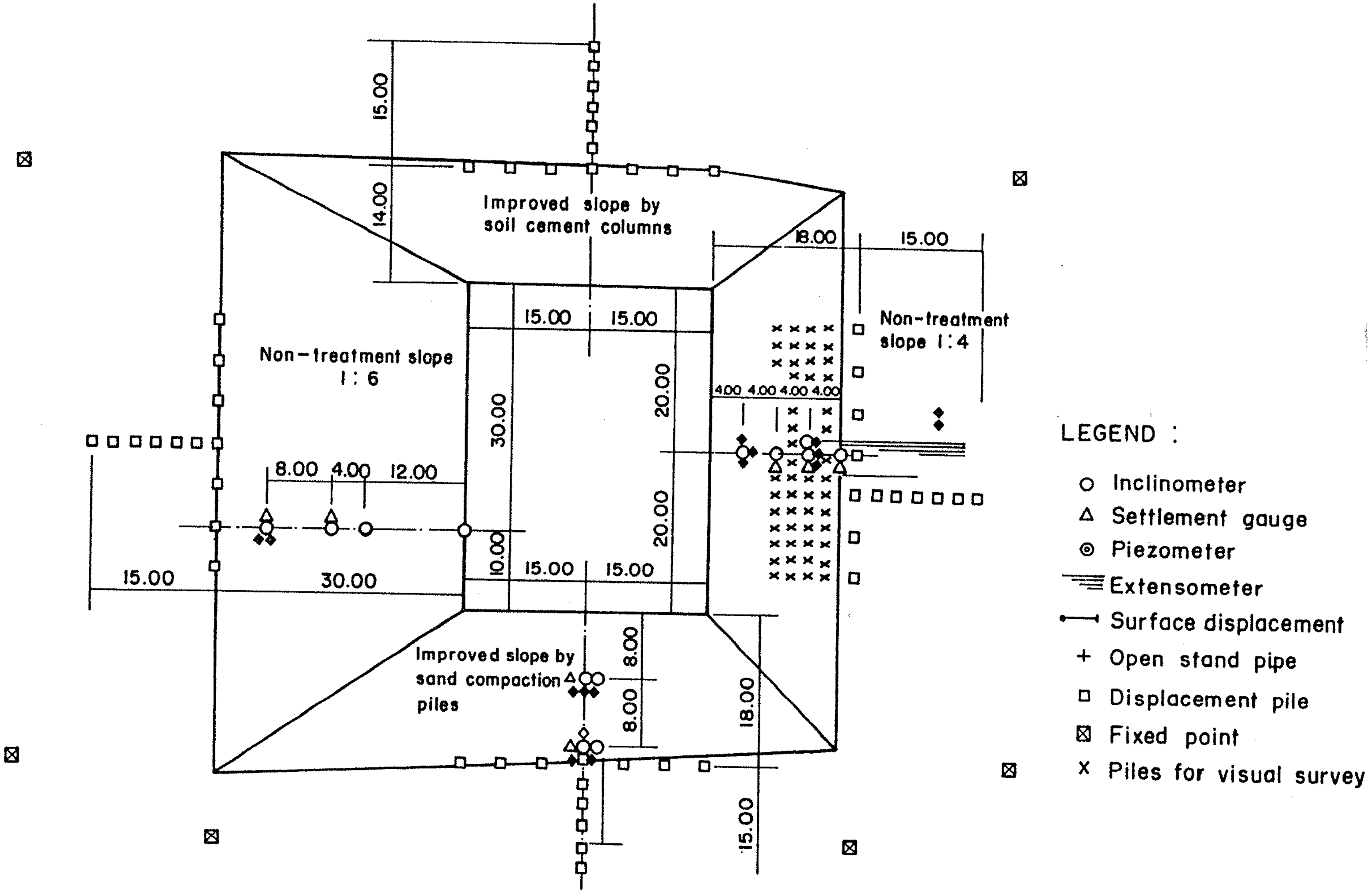
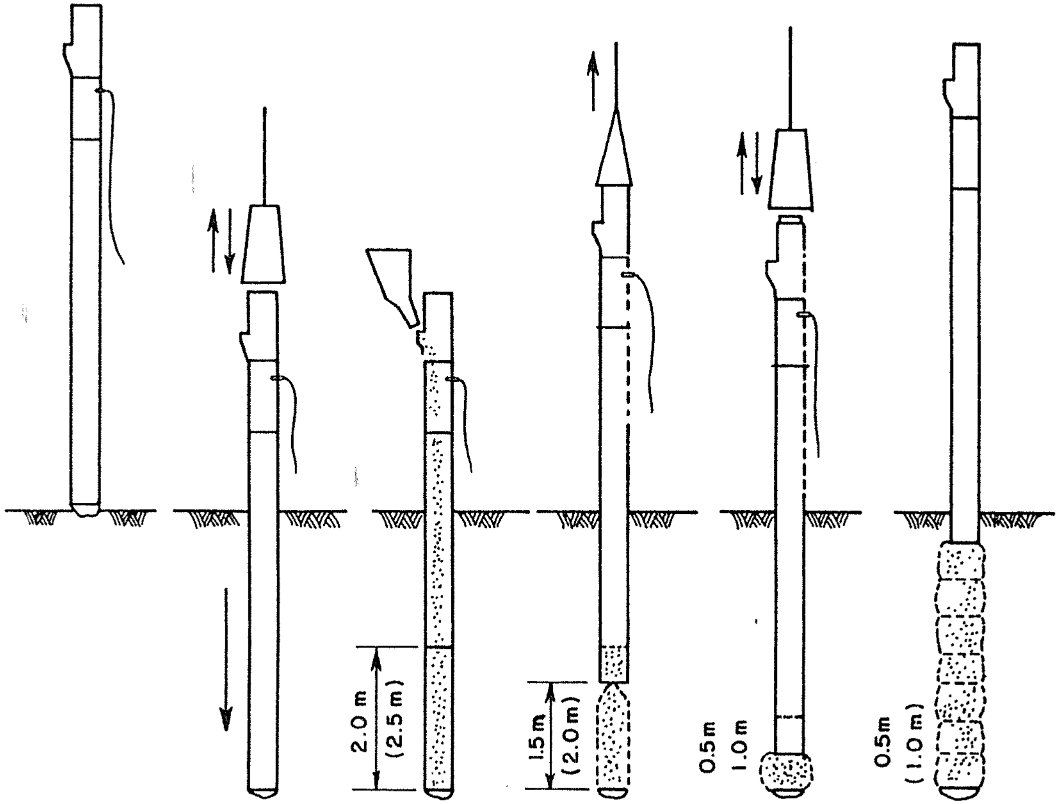


Fig. 5.22 Plan of Excavated Slope Facility



1. A sand bag is put on the spot where the sand compaction pile is to be constructed
2. Pile the casing pipe into the ground with the stone hammer
3. Pour the sand in the casing 2.0 m in height
4. Pull up the casing 1.5 m giving compressed air
5. Compact the casing with stone hammer until the casing sinks 1.0 m
6. Repeat the procedure 3 to 5

Fig. 5.23 Installation of Sand Compaction Piles

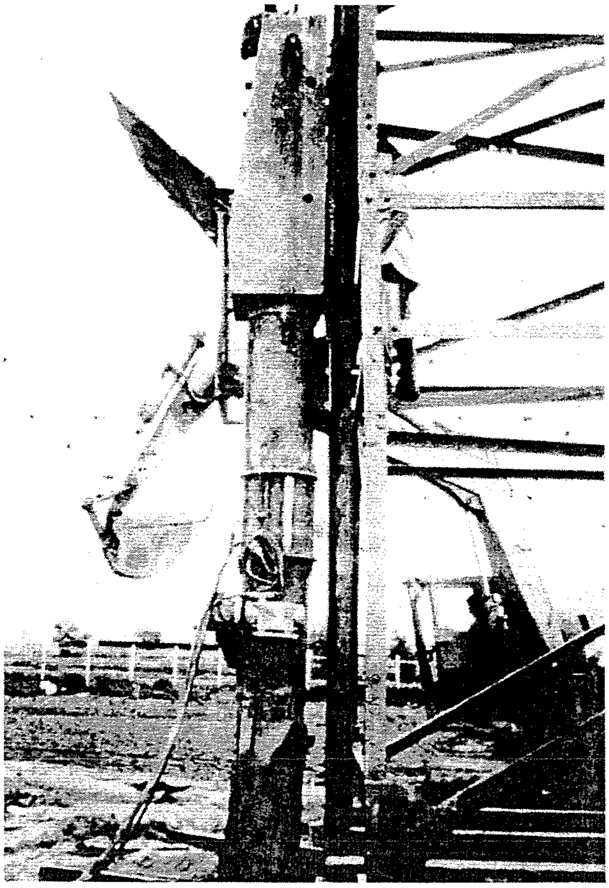


Fig. 5.24 Construction of Sand Compaction Pile on Soft Bangkok Clay By Sand Compaction Method

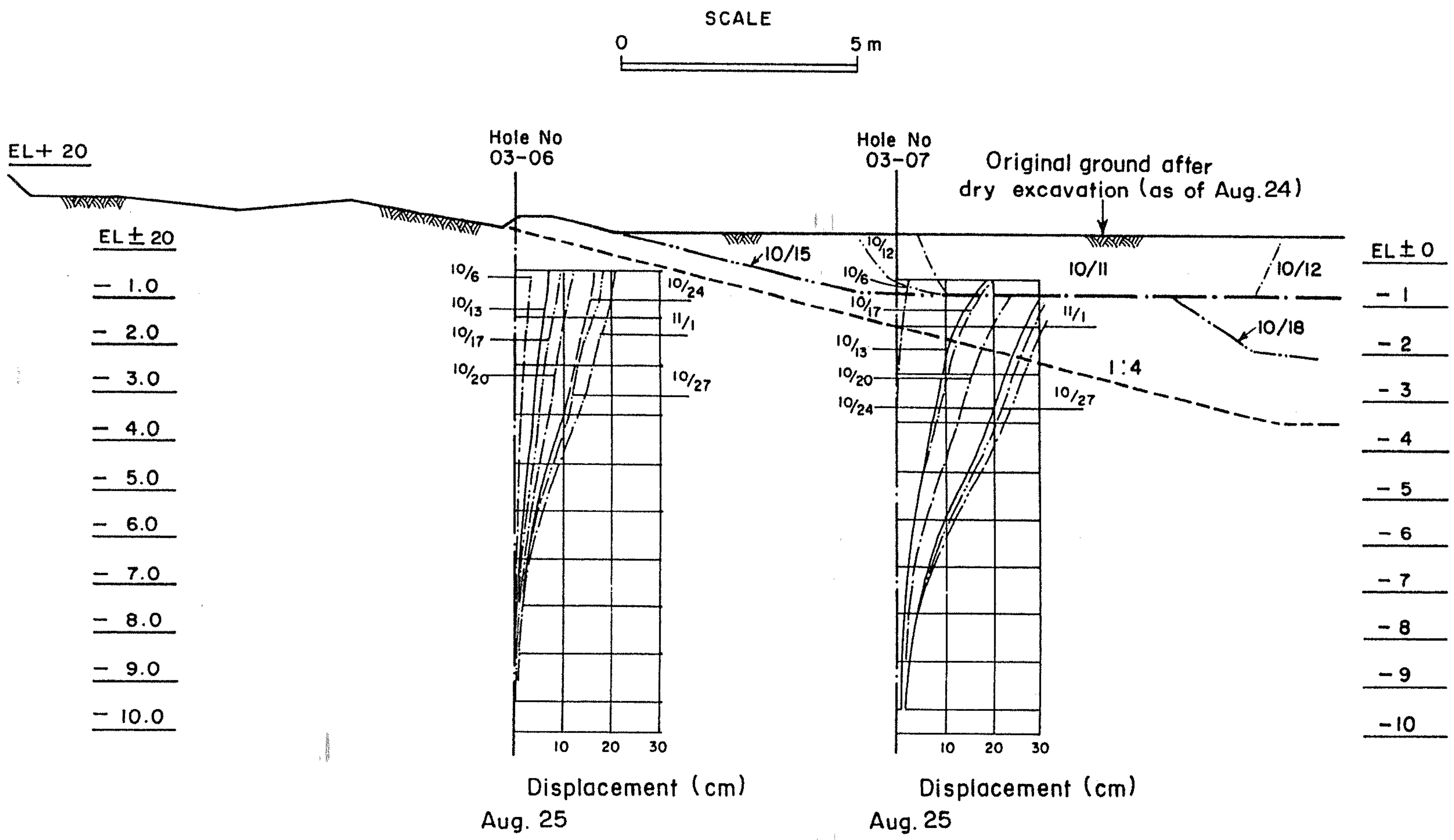


Fig. 5.25 Observed Lateral Displacements at Bang Bo Site

was stopped had a gradient of 1V:6H with maximum depth of 3.5 m.

5.15 EVALUATION OF LOCALLY AVAILABLE MATERIALS FOR SAND COMPACTION PILES (SCP)

In Thailand and in the Southeast Asian Region, the problem of the SCP method is the selection and identification of alternative types of sand suitable for SCP construction which are less costly, though may be of less quality, to be competitive. Clean sands are quite expensive. While clayey to silty sands cost half as much. Based on previous works of Nutalaya et al. (1984), Tjakrawiralaksana (1980) and Selvakumar (1977), the probable sources of sandy materials in the Lower Central Plain of Thailand (Fig. 5.26) can easily be identified. The extent of these deposits varies in depth, from 2 to 15 m, and covers areas of several square kilometers. The areas investigated were located within 100 km from Bangkok considering the most economical maximum distance of hauling of the materials for SCP to be competitive.

In order to evaluate the physical and engineering properties of the materials for SCP, sand samples were collected from the Ayutthaya, Chonburi, Kampengsen, and Ratchaburi areas. The materials obtained from former three locations were sandy soils, whereas the latter was laterite. The possibility of using lateritic residual soil as SCP construction material is considered since the material is readily available and cheap in Thailand.

Table 5.4 summarizes the laboratory test performed in order to determine physical and engineering properties of the materials (Bersabe, 1992). The grain size distributions of the four materials are shown in Fig. 5.27 with the suitable range of gradation for SCP materials which is used in Japan and the U.S. as reported by Fudo Construction (1991) and indicated by the broken lines.

Constant volume direct shear and triaxial (CID) tests were conducted to determine the strength properties and stress-strain behavior of the samples. Nine CID tests were performed for each type of sample corresponding to 3 different relative densities (60%, 70%, and 90%) and 3 different confining stresses (5, 10, and 15 tsm). Except lateritic soil, the other three samples were used for direct shear test, 9 tests per each sample using the same relative density levels used in CID tests and under three different normal stresses (5, 10, and 15 tsm). To achieve the desired relative densities in the sample preparation, a combination of equal-volume and undercompaction, as reported by Dennis (1988), was employed.

Table 5.4 also shows the results of the direct shear and triaxial (CID) tests along with the permeabilities of the samples. The Ayutthaya and Kampengsen sands indicated a significant increase in the angle of internal friction with increasing relative densities under consolidated drained conditions. The laterite showed only a slight increase in internal friction angle which was probably caused by grain crushing during the shearing. This difference is not so prominent in Chonburi material due to its fine grained nature (18% passing the No. 200 US Standard sieve). Generally, laboratory experiments indicate that the shearing strength and internal friction angle increase with increase in relative density of the sample. However, according to Vesic and

Clough (1968), at higher pressures, particles of cohesionless soils tend to crush each other thereby reducing the angle of shearing resistance. The behavior of the lateritic soil agrees with this. Table 5.4 also shows that the permeability of the samples decreases with the increase of relative density.

Large scale SCP model tests were performed by Leong (1992) and Cahulogan (1993) in the Asian Institute of Technology's Geotechnical Engineering Experimentation Site (Fig. 5.28). The schematic diagram of the model test system is illustrated in Fig. 5.29. A 0.90 diameter, 2.5 m high SCP steel mold was used to carry out model tests at three different relative densities (approximately 60%, 70%, and 90%). Each sample was tested in three set-ups corresponding to the desired relative density. A molding water content of about 5% was maintained as the sample was placed and compaction was made by layers using a compaction vibration and a tamper. The saturation process was provided through a pressure tank filled with water while carbon dioxide gas was used to saturate the sample. The samples were consolidated at three different pressure levels by using dead loads which were also used to apply confining pressure. Standard Penetration Tests (SPT) and Cone Penetration Tests (CPT) were performed for each set-up. Constant head permeability tests were also performed in every set-up to observe the drainage behavior.

Table 5.4 Internal Friction Angles and Permeabilities of the Samples

SAMPLE	Relative Density (%)	Friction Angle (ϕ) deg. (CID)	Friction Angle (ϕ) deg. (Direct Shear)	Permeability (x 10^{-4}) (cm/sec)
Ayutthaya	90	38.7	26.6	0.87
	70	34.6		4.02
	60	32.0	24.5	5.30
Chonburi	90	35.5	33.4	6.24
	70	35.0	31.0	12.80
	60	34.1	28.8	14.68
Kampengsen	90	39.0	30.1	2.50
	70	36.5	26.6	7.97
	60	34.6	25.6	16.5
Laterite	90	37.1		2.66
	70	36.2		8.35
	60	34.9		17.53

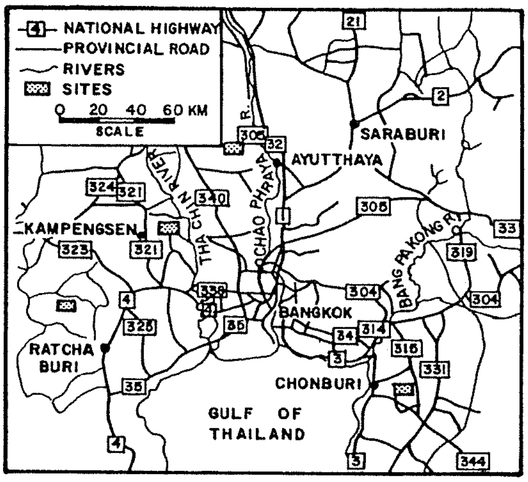


Fig. 5.26 Map of Areas Explored in the Lower Central Plain of Thailand

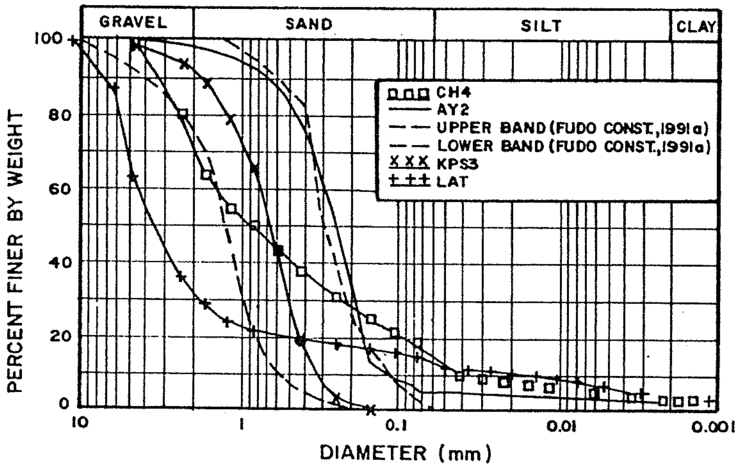


Fig. 5.27 Grain Size Distribution Curves

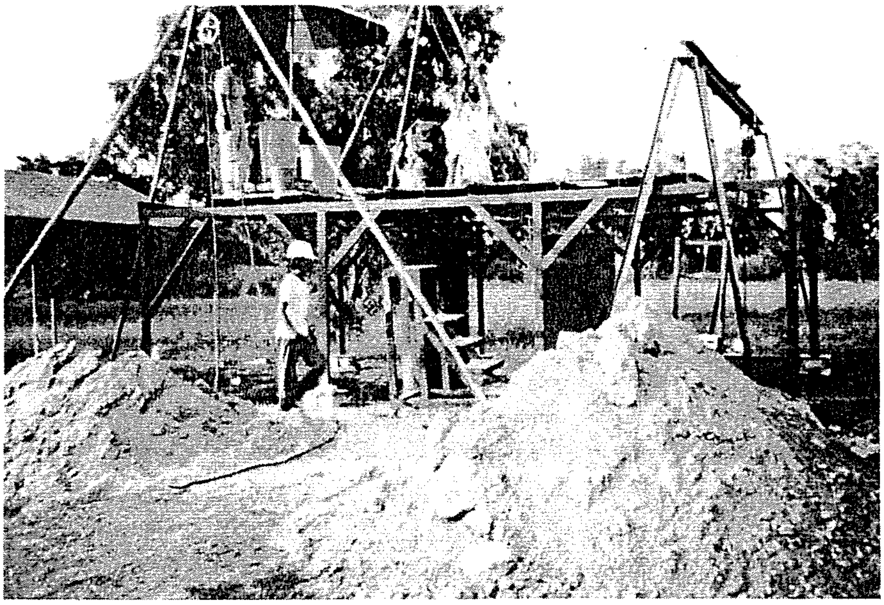


Fig. 5.28 SCP Model Tests in AIT

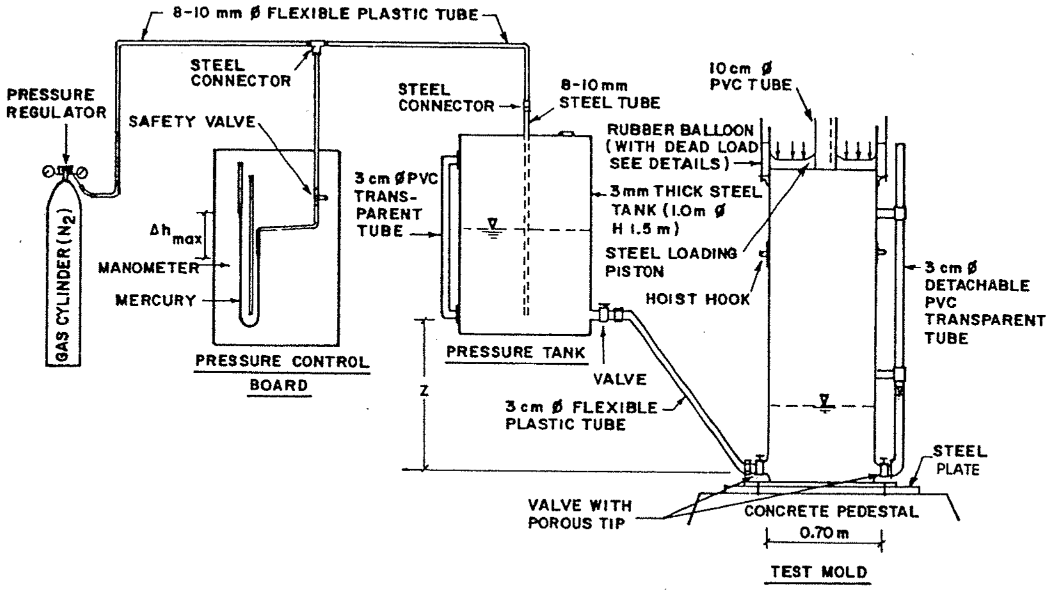


Fig. 5.29 Schematic Diagram of Model Test System

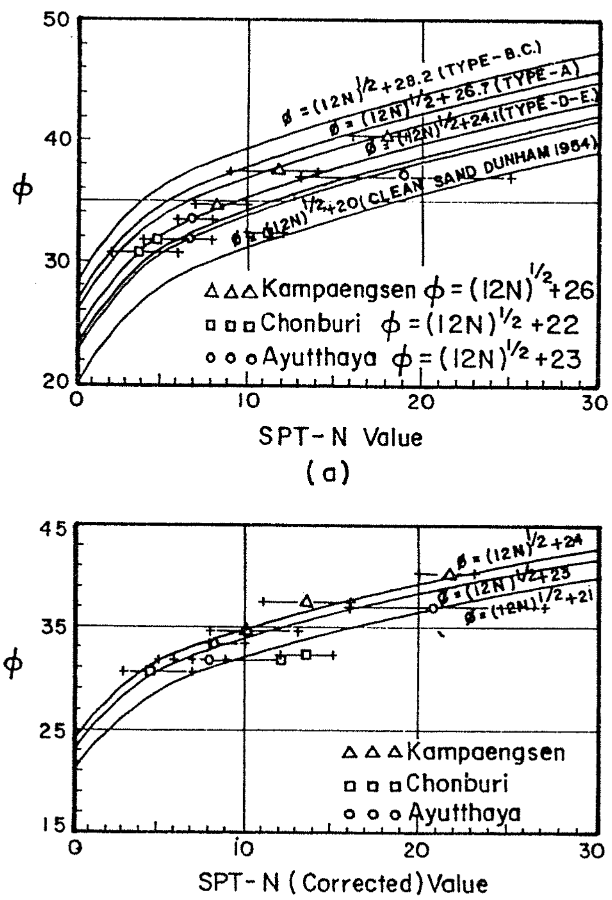


Fig. 5.30 Relationship Between Internal Friction Angle and SPT-N Values

The SPT-N values of the samples used in the model tests at 3 different relative densities (60%, 70%, and 90%) and three different vertical stresses are summarized in Table 5.4. Two sets of tests were conducted for each specimen for better comparison of the results. Raw SPT-N values from the model tests were corrected using the empirical equation of Gibbs and Holtz (1957). The SPT-N values significantly increased corresponding to the increase in relative density. Correlations of the internal friction angle with SPT-N values from the model tests indicated that the materials from Ayuthaya and Kampengsen followed the general equation of Dunham (1954) together with the SCP materials in Japan indicated as Type A to E in Fig. 5.30a,b.

The Dutch cone test results in the model tests showed good correlation between the internal friction angle and cone resistance which generally agree with the graphical correlation recommended by Meyerhoff (1974) and Robertson and Campanella (1983). Figures 5.31 and 5.32 show these agreements, respectively.

Based on the preliminary results of the laboratory and model tests, Ayuthaya and Kampengsen materials are recommended as suitable SCP construction materials. Their fine contents (percentage passing the No. 200 U.S. standard sieve) must be limited to about 8% for them to function well. However, the Ayuthaya material is recommended to undergo reliable filter characteristics in as much as it has finer grains than the Kampengsen material. Chonburi material can also be a good SCP construction material if it will undergo washing which will limit its fine contents to about 5% to 8%.

The lateritic soil showed quite acceptable properties needed for SCP design. However, uncertainties may arise, especially that its swelling potential was not considered. No further tests and investigations were made.

Since SPT has been employed widely to determine the actual performance of SCP, the following mathematical equations derived from the model test results can be used to assess the internal friction angles of the materials from SPT-N values as follows:

<u>From raw SPT-N values</u>:

Kampengsen:

$$\phi = (12N)^{1/2}+23.3 \tag{5.33}$$

Chonburi:

$$\phi = (12N)^{1/2}+22 \tag{5.34}$$

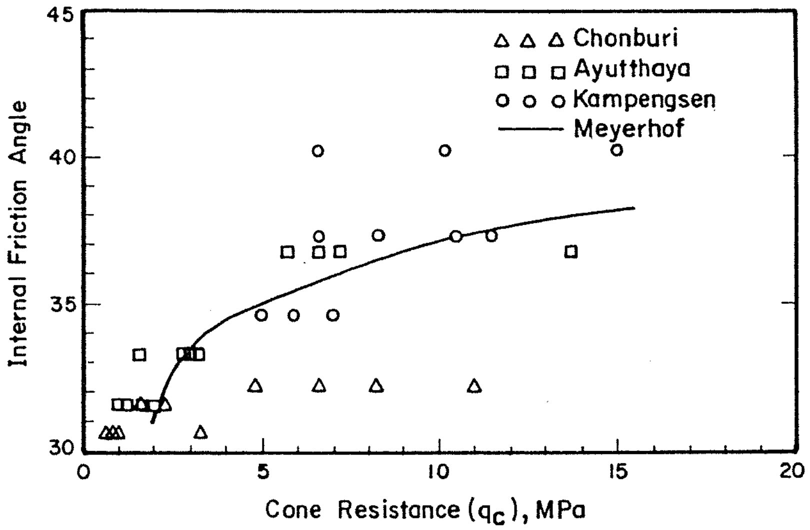


Fig. 5.31 Correlation of Internal Friction Angle and Cone Resistance

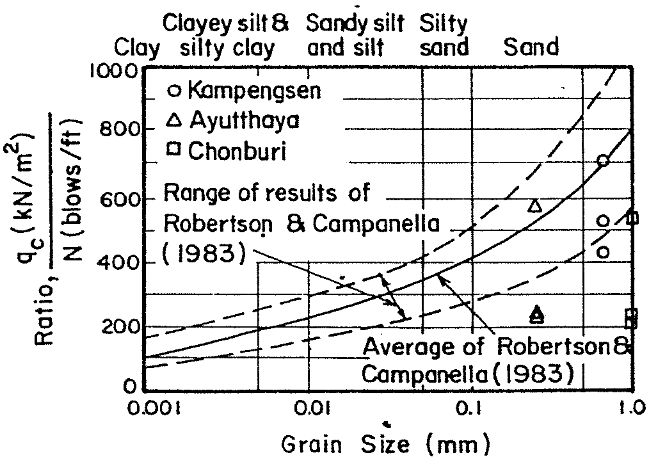


Fig. 5.32 Variation of q_c/N Ratio with Mean Grain Size for Sand Materials

Ayuthaya:

$$\phi = (12N)^{1/2}+22.8 \tag{5.35}$$

The regression equations using corrected SPT-N values are as follows:

From Corrected SPT-N values:

Kampengsen:

$$\phi = (12\acute{N})^{1/2}+26 \tag{5.36}$$

Chonburi:

$$\phi = (12\acute{N})^{1/2}+23.4 \tag{5.37}$$

Ayuthaya:

$$\phi = (12\acute{N})^{1/2}+22.4 \tag{5.38}$$

These correlations will facilitate the design and quality control of SCP construction.

5.16 GRANULAR PILES IN COMBINATION WITH OTHER SOIL IMPROVEMENT TECHNIQUES

At Ebetsu in Hokkaido, Japan, two full scale test embankments were constructed with different soil stabilization methods in addition to the test embankment without treatment (Aboshi and Suematsu, 1985). One of the test embankments was stabilized with sand drains and steel sheet reinforcements while the other was stabilized with sand compaction piles. At an embankment height of 3.5 m, the test embankment without treatment collapsed with substantial deformations on the subsoil and cracks in the embankment fill. While the embankments treated with sand drains and steel reinforcements and with sand compaction piles were completed up to 8 m high without failures. The test embankment stabilized by sand compaction piles yielded a stress concentration factor of 3 which is in close agreement with the observations of Bergado et al. (1988) on soft Bangkok clay improved with granular piles having a stress concentration factor ranging from 2 to 5. Based on the results of these test embankments, it is envisioned that the use of granular piles and steel grids reinforcements could be a good combination for embankments of soft Bangkok clay. The steel grids reinforcements help minimize the lateral spreading of the embankment and provide steeper side slopes or even vertical sides. The granular piles provide the reduction of settlements as well as the increase in strength and bearing capacity of the soft ground foundation. An example for such an application scheme is on transition units

Table 5.5 Raw and Corrected SPT-N Values from Model Tests

Sample	γ_t	Vertical Pressure (6.2 tsm)				Vertical Pressure (8.5 tsm)				Vertical Pressure (10 tsm)			
		N		N'		N		N'		N		N'	
		Set 1	Set 2	Set 1	Set 2	Set 1	Set 2	Set 1	Set 2	Set 1	Set 2	Set 1	Set 2
Ayuthaya	60	5	6	6	8	8	8	9	9	4	5	4	5
	70	6	8	8	10	7	8	8	9	8	9	9	10
	90	15	13	18	16	20	25	22	27	15	33	10	14
Choburi	60	2	3	3	4	4	6	4	7	4	5	4	5
	70	5	5	6	6	5	6	5	7	5	5	5	5
	90	10	12	13	15	13	11	14	12	14	12	15	13
Kampengsen	60	8	10	10	13	8	7	9	8	10	5	11	5
	70	9	13	11	16	14	11	15	12	15	9	16	10
	90	19	16	23	20	25	20	27	-	23	25	18	20

to bridges and viaducts as shown in Fig. 5.33. As an alternative of using expensive ideal materials for embankment fill such as sand, savings in the construction costs can be realized by using cheaper, locally-available, cohesive-frictional soil with more than 18% having particle size diameter lower than 0.74 mm. Extensive research has been done on steel grids reinforcements with poor quality backfills consisting of weathered clay, lateritic soils and clayey sand (see Bergado et al. 1990a). However, the combination scheme of granular piles and steel grids reinforcements to improve the ground and embankment fill, respectively, must be studied through full scale field prototype so that their effectiveness on soft and subsiding Bangkok clay would be proven and the actual reduction of lateral spreading can be measured.

5.17 CONCLUSIONS

The current state-of-the-art on granular pile scheme has been discussed but there are still some loopholes which need to be studied for better understanding of this ground improvement method. Among these are the influences of the method of construction, characteristics of pile materials, better estimation of stress concentration factors, stress distribution with depth and time, improvement factors as well as the effect of overlying surcharge consisting of mechanically stabilized (reinforced) embankment. It was indicated that there is a need for additional research to improve the design methods and develop a complete understanding of the mechanics of granular pile behavior. More full scale and model tests with extensive instrumentations in combination of finite element model studies should be carried out to shed light to the uncertainties and improve confidence, especially when the scheme is applied to the soft and subsiding ground.

Already, a test embankment on granular piles has been constructed at AIT to evaluate and investigate the applicability of such a scheme on the soft and subsiding Bangkok clay. The performance of this test embankment has been described in the preceding section. The improvement factors on bearing capacity, settlement and gain in strength have been determined and indicated substantial values. Based on the results of these studies as well as in Japan, it was suggested to use granular piles in the subsoil in combination with mechanically stabilized earth (MSE) embankments using grid reinforcements as one alternative for ground improvement at the approach embankments to bridges and viaducts. The implementation of this ground improvement scheme was postponed mainly due to the lack of available data regarding its actual performance. Therefore, it is deemed necessary to construct a full scale field prototype so that the effectiveness of granular piles, possibly combined with other soil improvement techniques, as an alternative scheme for ground improvement on approach embankments to bridges and viaducts would be proven in the soft and subsiding environment of Bangkok. Finally, the use of cheaper materials for SCP construction has been found to be viable and economical alternative.

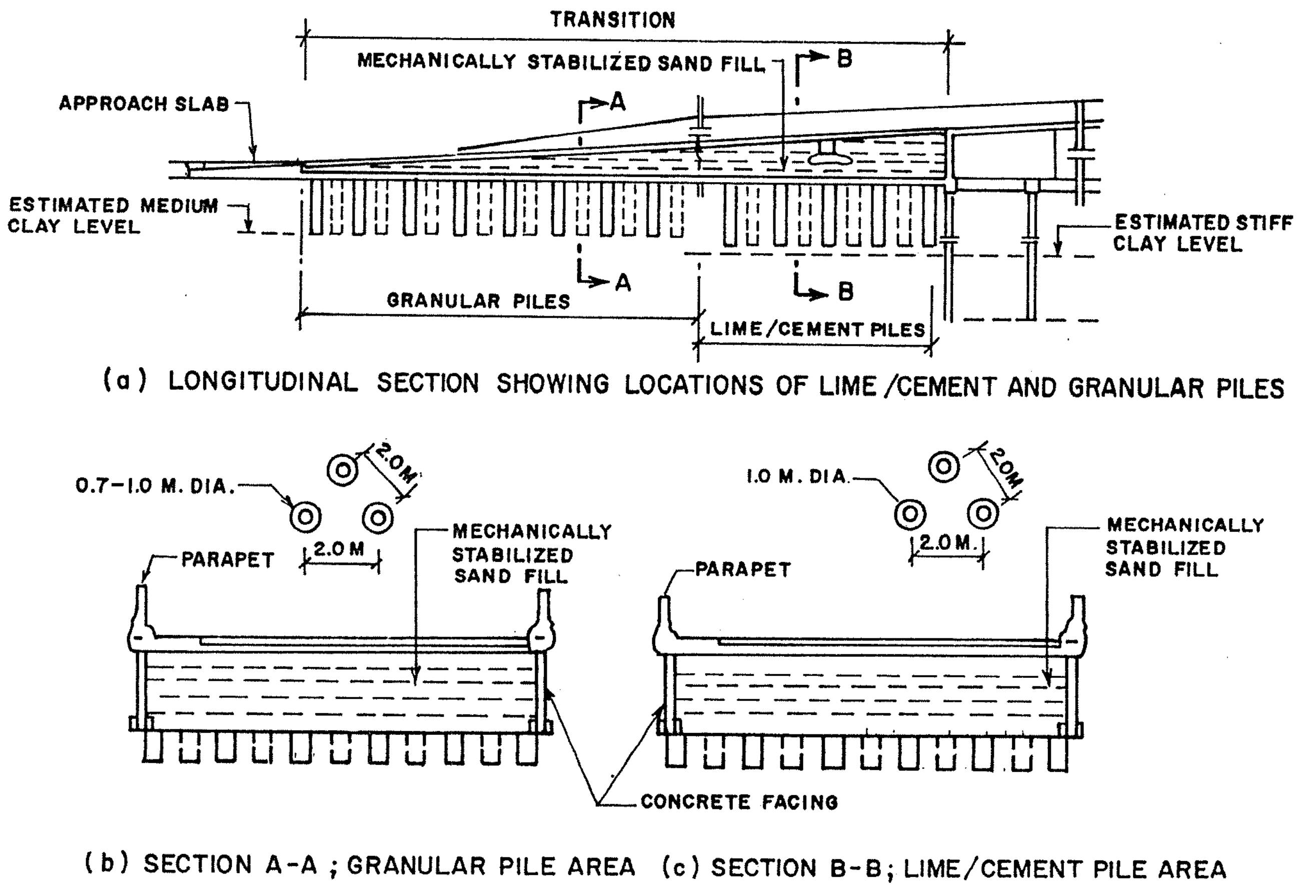


Fig.5.33 Granular Piles with Mechanically Stabilized Embankment

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APPENDIX
(Notations for Tables 5.1 and 5.2)

A_s	: cross-sectional area of sand pile
B	: width of loaded area
C_c	: compression index of clay
C_o	: original undrained shear strength of clay
D_f	: depth of foundation
E	: modulus of elasticity
F'_c, F'_q	: cavity expansion factors
H	: thickness of layer
K	: earth pressure coeffficient applying to the load increments
K_{as}	: active earth pressure coefficient of sand pile
$[K_c^{(m)}]\{\Delta\sigma^{(m)}\}$	: vector of corrections due to yielding of the pile and/or clay; these corrections are treated as an additional external load
$[K_E]$	: elastic stiffness matrix
K_o	: coefficient of earth pressure at rest
K_{pc}	: soil coefficient of passive earth pressure
L	: length of sand pile
N_c, N_γ, N_q	: dimensionless factors which depend on properties of soil and pile material and area replacement ratio
$(P_o)_{vc}$	: initial vertical stress in the clay
R	: settlement reduction factor
S_t	: settlement of composite foundation
W	: width of equivalent granular pile strip
Z	: depth from surface of composite foundation
a_s	: area replacement ratio
d_s	: pile diameter
e_o	: initial void ratio
m	: iteration number
m_v	: modulus of volume compressibility
n	: stress concentration factor
q_o	: overburden pressure
q_s	: bearing capacity of soft soil expressed as $(2/3)C_oN_c$
q_{ult}	: ultimate bearing capacity
γ_s, γ_c	: unit weight of sand and clay, respectively
$\{\Delta F_{DN}\}$	: vector of incremental nodal forces at the dual nodes along the pile clay interface
$\{\Delta F_E\}$	: vector of incremental nodal forces due to applied tractions (usually applied along the top of the sand pile)
$(\Delta P)^*_{vc}$	: effective vertical stress increase in the clay averaged over the horizontal projected area of clay
$(\Delta P)^*_v$	: effective vertical stress increase in the clay averaged over the horizontal projected area of unit cell
$\{\Delta\sigma^{(m+1)}\}$	: vector incremental deflections
ε_v	: vertical strain (same for sand and clay)
θ	: vertical angle of sliding surface at each sand pile
μ	: Poisson's ratio
μ_c	: reduction in stress coefficient of clay
σ	: vertical stress
σ_{ro}	: initial radial stress along the granular pile
σ_z	: overburden pressure at depth z
τ	: shear resistance of composite foundation
ϕ_s	: angle of internal friction
Ψ	: angle between the assumed failure surface and foundation